

## CHAPTER 8

### TRANSMISSION LINE DESIGN

**8-1. General.** This chapter includes procedures specifically related to transmission lines of 12 inches diameter and larger. Service connections are usually not permitted. Interconnections with the distribution system piping should be held to a minimum and are usually over 1,000 feet apart.

**8-2. Design procedures.** Transmission line design shall include the following procedures.

*a. Layout.* The new line will be designed to fall within existing utility or street right-of-way where available. The price of acquiring easements through private property must be considered as part of the alternative cost analysis. Easements must be wide enough to allow for initial construction and future maintenance. Installation close to physical features, such as buildings or other utilities, which would cause construction problems or future access problems for maintenance should be avoided. A set of plan and profile drawings shall be prepared which shall show as a minimum the following information:

- (1) Survey base line with physical control points.
- (2) Easements, rights-of-way, streets, and construction limits, etc.
- (3) Existing physical features such as buildings, fences, structures, utilities, trees and drainage systems.
- (4) Existing, and proposed if applicable, ground elevations along the centerline of the pipe shall be shown on the profile.
- (5) In plan, the proposed pipeline and its relationship to the survey base line.
- (6) In profile, the centerline elevation of the proposed pipeline.
- (7) Beginning and ending points of the pipeline and all appurtenances.
- (8) Construction details of the pipeline, connections, appurtenances, tunneling, bedding and surface restoration, etc. Typical information shown on plan and profile drawings is illustrated in figure 8-1.

*b. Diameter vs pumping costs.* Pump costs are dependent upon initial cost and horsepower requirements. The total dynamic head of the system is the sum of the suction lift, discharge head, friction head

and velocity head, and is represented by the following equation:

$$\text{TDH} = H_S + H_D + H_F + \frac{V^2}{2g} \quad (\text{eq 8-1})$$

where: TDH = total dynamic head

$H_S$  = suction lift

$H_D$  = discharge head

$H_F$  = friction head

$\frac{V^2}{2g}$  = velocity head

The most economical pipe diameter in a pumped system is determined by comparing pumping costs for various sizes of pipe. Only standard pipeline sizes should be considered. In order to hold friction losses to a minimum and to reduce the possibility of severe waterhammer, the diameter should be sized so that the velocity is 4 feet per second (fps) or less. Under special circumstances when approved by the appropriate authority, the maximum design velocity may exceed 4 fps.

*c. Hydraulic calculations.* The Hazen-Williams formula is often used to compute flow characteristics. Also, the Darcy-Weisbach equation is used in some computer programs. Depending on requirements, one of the following forms of the Hazen-Williams equation is used:

$$V = 0.55 C D^{0.63} S^{0.54} \quad (\text{eq 8-2})$$

$$Q = 0.433 C D^{2.63} S^{0.54} \quad (\text{eq 8-3})$$

$$S = \frac{2.32Q}{C D^{2.63}} 1.85 \quad (\text{eq 8-4})$$

$$V = 1.318 C R^{0.63} S^{0.54} \quad (\text{eq 8-5})$$

where:

$V$  = velocity of flow in feet per second

$C$  = coefficient of roughness

$D$  = diameter of pipe in feet

$S$  = head loss in feet per foot of length

$Q$  = flow in cubic feet per second

$R$  = hydraulic radius in feet

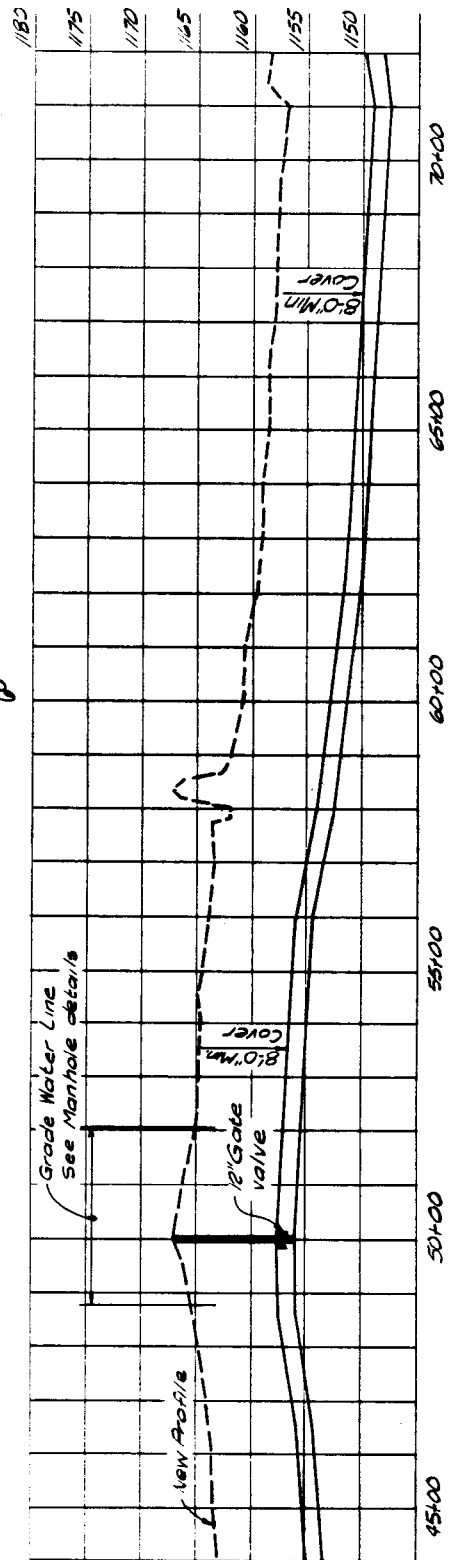
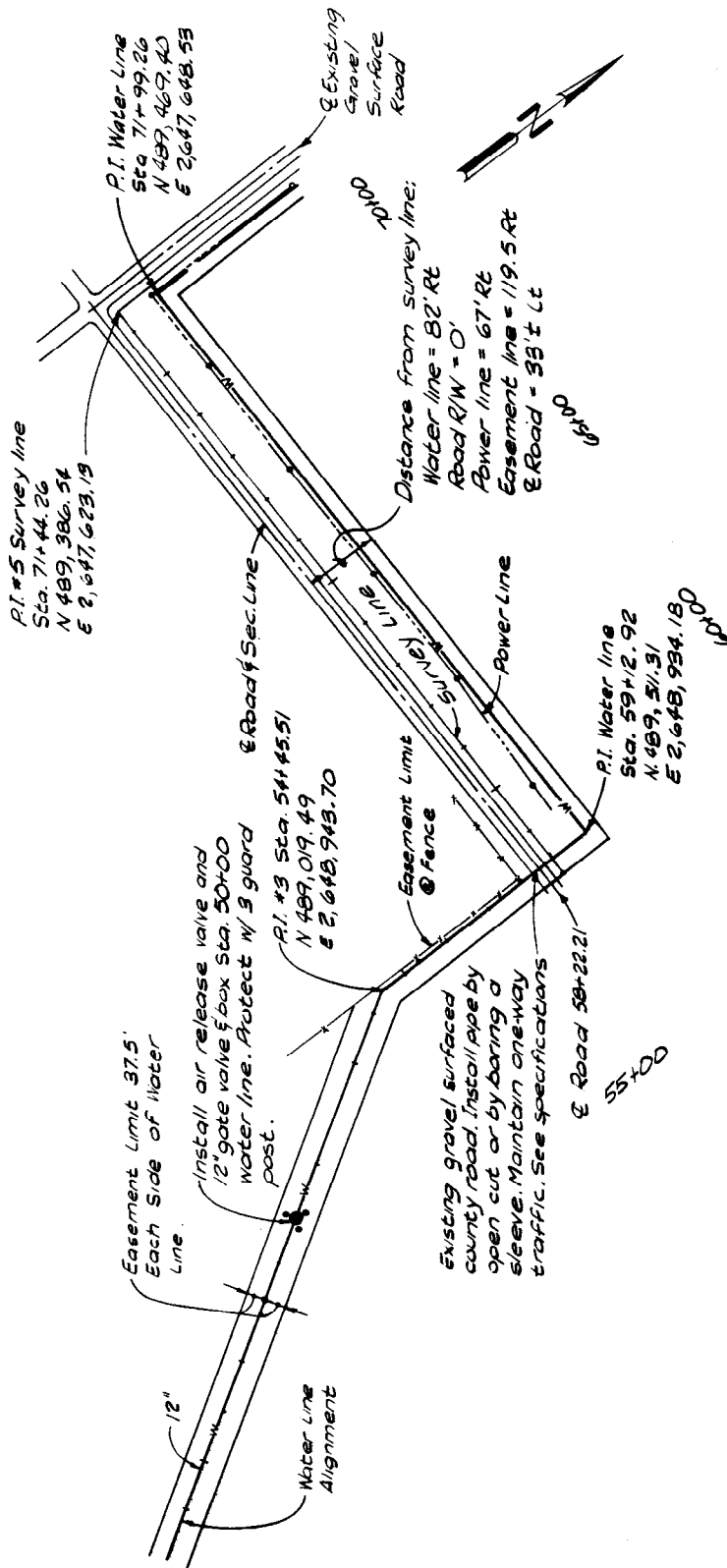


Figure 8-1. Typical plan and profile drawings.

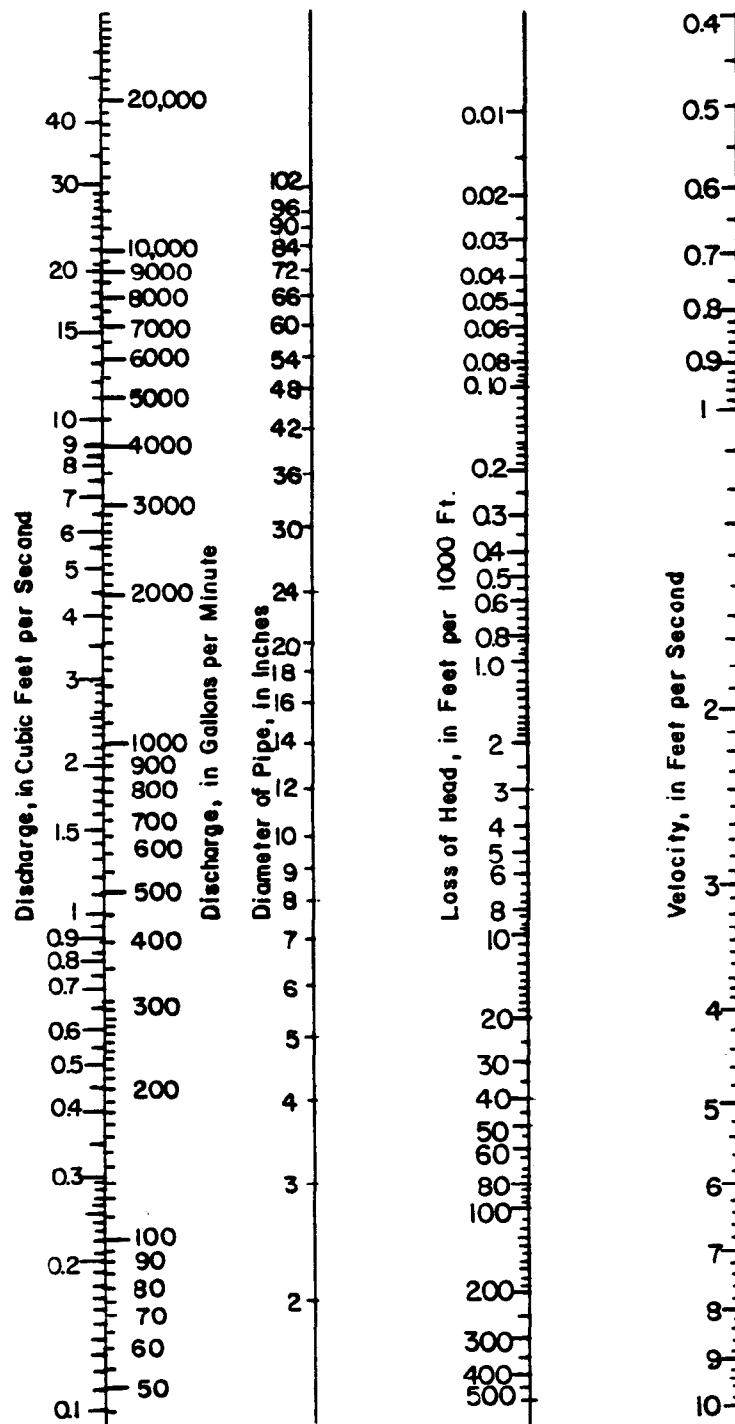


Figure 8-2. Nomograph for Hazen-Williams formula in which  $C = 100$ .

Values of C are 140 for smooth lined steel pipe, very smooth concrete pipe, cement lined ductile iron pipe and asbestos-cement pipe; 130 for ordinary ductile iron pipe in good condition; 110 to 120 for ductile or cast-iron pipe in service 5 to 10 years; 100 for older cast-iron pipe; and 40 to 80 for old cast-iron or steel pipe which is severely tuberculated or any pipe with heavy deposits. A quick solution for the equations may be found by use of the nomograph in figure 8-2. It is prepared from the Hazen-Williams formula by using  $C = 100$ . For larger or smaller values of C, the discharge or velocity obtained from the nomograph is multiplied by the ratio of the given value of C to 100. If the discharge or velocity is given, it should be multiplied by the ratio of 100 to the known value of C before the nomograph is used.

**EXAMPLE 8-1.** By using figure 8-2, determine the discharge, in cubic feet per second (cfs), from a 12-inch pipe for which  $C = 120$  when the loss of head is 5 feet per 1000 feet.

**Solution:** The discharge corresponding to the given diameter and loss of head for a value of  $C = 100$  is found first. A straightedge passing through 12 on the diameter line and 5 on the loss-of-head line will intersect the discharge line at 2.5 cfs. Therefore, the discharge is:

$$2.5 \times \frac{120}{100} = 3.0 \text{ cfs}$$

**EXAMPLE 8-2.** A 30-inch pipe for which  $C = 130$  is to discharge 10,000 gallons per minute (gpm). By using figure 8-2, find the loss of head per 1000 feet of pipe.

**Solution:** The first step is to determine the discharge that corresponds to a value of  $C = 100$ . This is  $10,000 \times 100/130 = 7,692$  gpm. A straightedge through 30 on the diameter line and 7,692 on the discharge line intersects the loss-of-head line at 2.0 feet per 1000 feet.

Valves, bends, and other fittings in a pipeline and sudden enlargements or contractions cause loss of head. If a valve is partly closed, there is greater resistance to the flow and greater loss of head. Table 8-1 shows the loss in pipe fittings and appurtenances, expressed as equivalent lengths of straight pipe as a multiple of the diameter, due to various valves, fittings, contractions, and enlargements.

**d. Internal pressure.** The internal pressure is the difference in elevation between the conduit and the hydraulic grade line (HGL). The pressure at the beginning of a main may be generated by a pump, reservoir elevation or connection from another pipeline. Pressure losses are due to friction, bends and fittings, and changes in elevation. These are meas-

ured above a horizontal datum plane as shown on figure 8-3.

**EXAMPLE 8-3.** Assuming steady state conditions the pump in figure 8-3 delivers 5,000 gpm at a discharge pressure of 140 feet of head. The pipeline is 24-inch diameter,  $C = 100$ , and is 3,625 feet from pump to reservoir. Calculate the internal pressure at Station 20 + 00.

**Solution:** From figure 8-2, the head loss is found to be 2.7 feet per 1,000 feet. Assume other losses are negligible.

$$h_L = 2 \times 2.7 = 5.4 \text{ ft.}$$

Elevation at pump =	230.0	
Discharge pressure =	+ 140.0	(60.0 psi)
Elevation of HGL at pump =	370.0	
Less $h_L$ =	- 5.4	
Elevation of HGL at Station 20 =	365.6	
Pipe Elevation =	- 310.0	
Pressure =	55.6ft.	(24.1 psi)

**e. Pipe materials.** The materials allowed for use are steel, ductile iron, reinforced concrete, asbestos-cement, glass fiber reinforced and polyvinyl chloride pipe. Mains must be designed for the maximum internal working pressure plus an allowance for waterhammer. Design must include external stress due to earthfill and superimposed loads. Reductions in wall thickness at isolated regions of lower pressure or reduced external stress will not be made due to the possibility of construction personnel installing these sections in the wrong sections of the line. Pipe shall be designed and specified in accordance with applicable AWWA Standards, and specifically as indicated in table 5-3.

**f. Anchorage and Expansion.**

(1) Thrust restraint at bends and abrupt changes in direction is required. Certain types of pipe such as welded steel are designed as continuous conduits and some ductile iron joint systems are designed as restrained joints which may not require thrust blocking. When required, thrust restraint shall be designed in accordance with appendix C.

(2) Under empty conditions, lightweight conduit such as steel is bouyant and will float. This may occur if the water table is high enough even though the pipeline may be backfilled. If this is likely, the designer must add extra weight to the pipe. This

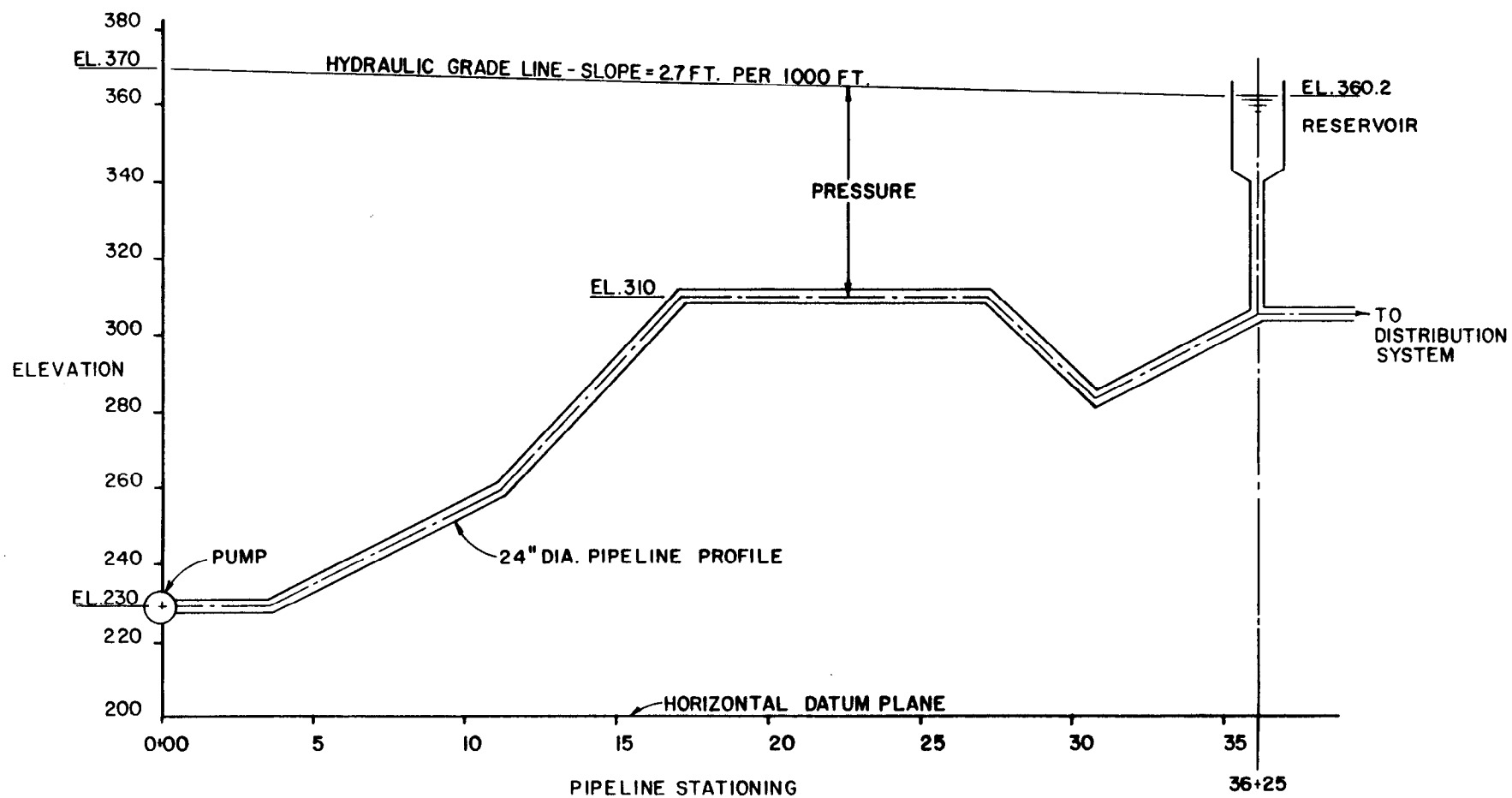
Table 8-1. Losses in pipe fittings and appurtenances

Description of Pipe Fitting or Appurtenances	Loss in Equivalent Length of Pipe Diameters (d)
Gate Valve	
3/4 Closed	900
1/2 Closed	160
1/4 Closed	35
Full Open	13
Angle Valve Open	170
Globe Valve Open	340
Swing Check Valve	80
E l b o w s	
90° Standard	
90° Long Radius	20
45° Standard	16
Tee Flow Through Run	20
Standard Tee Take-Off	75
Run of Tee Reduce One-Half	32
Sudden Contraction:*	
d/D = 0.25	15
d/D = 0.5	12
d/D = 0.75	7
Sudden Enlargement:*	
d/D = 0.25	32
d/D = 0.5	20
d/D = 0.75	19
Entrance to Basin	75

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\*For contractions and enlargements, d is diameter of smaller pipe and D is diameter of larger pipe; resistance is expressed in terms of d.

Figure 8-3. Illustration of pipeline pressure.



may be accomplished by the use of reinforced concrete collars, poured in place, and firmly anchored to the pipe.

(3) All pipe materials are subjected to expansion and contraction forces due to temperature changes. In colder climates, water temperature may vary from 32 degrees F. to over 80 degrees F., a range of over 50 degrees. Above ground installations, especially in hot temperature climates may also cause significant temperature variations. In bell and spigot joints, this effect usually may be neglected. However, the effects of expansion and contraction can be significant for long, straight, continuous pipelines. The designer shall include expansion joints designed for the pipe material and/or include appropriate stresses in the calculation for pipe wall thickness where required. However, it is not usually economical to increase the strength or thickness of the pipe wall as the sole means of resisting these stresses.

*g. Valving.* Sectionalizing valves are to be provided at all connections to the main. This includes pump discharge, distribution connections, fire hydrants, blowoffs, air valves and reservoir connections. Line valves are not usually required to be closer than one mile unless intermediate distribution connections are made. For larger size mains, the use of valves one standard size smaller than the pipeline is allowed as a cost-saving measure, provided that the velocity through the valve does not exceed 11 fps. Many large line valves have an integral by-pass arrangement. Valves may be the same as used in the distribution system, see paragraph 5-1. Valves shall be adequately designed for the actual internal pressure.

*h. Air-vacuum valves.* Air-release valves eliminate air pocket build up which causes a flow constriction and increased head loss. They are designed to expel air from a line during filling and close automatically when water reaches the valve. Vacuum valves are designed to allow air to enter the main when it is being drained. Also, vacuum valves are required to prevent the possible collapse of thin wall conduit which may be subject to a vacuum under certain conditions such as a break in the pipeline. Combination air release and vacuum valves are to be installed at the following locations:

(1) Peaks, where the pipe slope changes from positive to negative.

(2) Long relatively straight stretches at 1/4-to 1/2-mile intervals.

Air valves are to be sized to exhaust air at the pipe fill rate. Vacuum valves are to be designed to admit air at a rate equal to the flow generated by gravity.

Consult manufacturer's literature for capacity and performance data. These valves are to be installed in pairs to prevent problems due to failure at one of the valves.

*i. Blow-offs.* Blow-offs, with a drain to a disposal area, should be installed near low points and other suitable locations to facilitate draining the conduit and disposal of the water. Blow-offs will be designed with an air-gap to prevent contaminated water from backing up into the main.

*j. Hydrants.* Hydrants for fire fighting purposes are not normally installed on transmission mains. If they are, design should be as specified in paragraph 5-2. Hydrants may be installed to facilitate filling and disinfection. For this purpose, a hydrant may be located adjacent to each line valve.

*k. Access manholes.* Manholes for access to the inside of large mains facilitate the construction and inspection of pipelines large enough to be entered by a workman. The minimum pipeline size is usually 20-inch diameter. They are useful if located adjacent to air valves, blow-offs, and line valves. Access manholes are to be designed as pipeline tees and fitted with a bolted blind flange.

*l. Flow measurements.* The design should allow for measurement of the volume of flow in the main. This may be done by pitot tube which requires the installation of a 1-inch corporation in the top of the main, or by a venturi installed as part of the pipeline or other commercially available equipment or methods. Sufficient straight pipe without flow interruption shall be provided ahead of and following the point of measurement as required by the manufacturer of the device.

*m. External corrosion.* The design, if the same as for distribution mains, refer to paragraph 7-5. Also use references cited in paragraph 8-2e.

*n. Area restoration.* Upon completion of the pipeline, the trench will be backfilled and compacted to prevent settlement. The surface will be brought to grade to match existing or design elevations. In previously grassy areas, the surface will be seeded or sodded. In paved or sidewalk areas, restoration shall match the original surface treatment as close as possible.

**8-3. Filling procedures.** Water should be admitted to the new transmission main at the lowest available point and be allowed to fill the pipe slowly up to higher elevations. Each section of main between line valves shall be filled separately and checked before proceeding to the next section. The progress of water in the pipeline shall be carefully and continuously monitored. It is usually neither feasible

nor necessary to begin the filling process prior to the completion of construction of the entire pipeline.

a. On mains longer than a few thousand feet, where it would be unwieldy to continuously refer to construction drawings, a special profile drawing may be prepared at a smaller scale; e.g., 1" = 100' or 1" = 200'. This profile drawing should show pipeline stationing, all appurtenances and other major physical and design features.

b. Prior to commencement of filling operations, all blow-offs, access manholes, other appurtenances, and temporary construction features should be checked to make sure they are closed and sealed. Check to see that air-release valves are free of debris and the control valves are open. Fire hydrants may be opened for additional air release and flushing purposes.

c. The rate of fill shall be carefully monitored and controlled. Use of two-way radios is desirable on longer pipelines. The point of fill should be continuously manned and personnel should be prepared to close valves in the event of leaks or other problems.

d. Water may be admitted through line valves on smaller lines. On larger mains, hydrants, distribution connections, or bypass connections of 6-inch or 8-inch diameter should be used. In any case, the valves being used must be capable of being closed under the conditions of flow with full head on one side only.

e. Progress shall be monitored by checking air-release valves and flow from hydrants. Hydrants may be closed when full-barrel flow is achieved without pulsing or surging. Each appurtenance must be checked for leaks during the filling process.

f. The filling process does not have to be a 24-hour-around-the-clock operation. It can be stopped any time, and later resumed. However, it cannot be carried on without being continuously monitored.

g. Upon completion of the filling process, the connection used shall be closed and each appurtenance shall be checked for leaks. Each section of main between line valves shall be individually filled, checked and disinfected before proceeding to the next section, and before putting it into service. Test pressures shall be in accordance with applicable AWWA Standards for the type of pipe used and the design pressure.

**8-4. Syphons.** Under special circumstances, a pipeline may be designed as a syphon. In these cases, a section of the main will be above the hydraulic grade line and therefore is under negative pressure. This design condition requires the specific approval of the installation's Facilities Engineer. Certain design details require special attention.

a. *Pipe material and thickness.* The pipe material and wall thickness shall be specifically designed for negative pressure. This requires a rigid pipe wall. Thin wall pipe, such as steel, may collapse under these conditions if not specially designed for negative pressure.

b. *Air-vacuum valves.* Air release and vacuum valves would defeat the design of a pipeline syphon. Their use must be carefully considered and should be limited to reaches of positive pressure if at all.



## APPENDIX A

### REFERENCES

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#### Government Publications

##### Department of the Army, the Navy, and the Air Force

MIL-HDBK-1008

Fire Protection for Facilities,  
Engineering, Design, and  
Construction

##### Department of the Army and the Air Force

TM 5-810-5/AFM 88-8, Chap.4

Plumbing

TM 5-809-10/AFM 88-3, Chap. 13

Seismic Design for Building

TM 5-811-4/AFM 88-45

Civil Engineering Corrosion  
Control-Cathodic Protection  
Design

TM 5-813-1/AFM 88-10, Vol. 1

Water Supply: Sources and  
General Considerations

TM 5-813-3/AFM 88-10, Vol. 3

Water Supply: Water Treatment

TM 5-813-4/AFM 88-10, Vol. 4

Water Supply: Water Storage

TM 5-813-7/AFM 88-10, Vol. 7

Water Supply for Special Projects

AFM 85-21

Operation and Maintenance of  
Cross Connection Control and  
Backflow Prevention Systems

AFR 86-5

Planning Criteria and Waivers for  
Airfield

AFM 88-10, Chap. 6

Water Supply: Water Supply for  
Fire Protection

AFM 88-15

Air Force Design Manual—  
Criteria and Standards of Air  
Force Construction

#### Non-Government Publications

American Railway Engineering Association (AREA), 59 East Van Buren Street, Chicago, Illinois 60605

Manual for Railway Engineering (each chapter issued and dated separately)

American Society of Mechanical Engineers (ASME), United Engineering Center, 345 E. 47th St., New York, New York 10017

Boiler and Pressure Vessel Code and Interpretations:

Section VIII

Pressure Vessels, Division 1

American Water Works Association (AWWA), 6666 West Quincy Avenue, Denver, Colorado 80235

C101	Thickness Design of Cast-Iron Pipe
C150	Thickness Design of Ductile Iron Pipe
C200	Steel Water Pipe 6 Inches and Larger
C203	Coal-Tar Protective Coatings and Linings for Steel Water Pipelines - Enamel and Tape - Hot-Applied
C205	Cement-Mortar Protective Lining and Coating for Steel Water Pipe - 4 inch and Larger - Shop Applied
C209	Cold Applied Tape Coatings for Special Sections, Connections, and Fittings for Steel Water Pipelines
C300	Reinforced Concrete Pressure Pipe, Steel Cylinder Type, for Water and Other Liquids
C301	Prestressed Concrete Pressure Pipe, Steel-Cylinder Type, for Water and Other Liquids
C302	Reinforced Concrete Pressure Pipe - Noncylinder Type, for Water and Other Liquids
C401	Selection of Asbestos - Cement Distribution Pipe, 4 inch, through 16 inch, for Water and Other Liquids
C403	Selection of Asbestos-Cement Transmission and Feeder Main Pipe
C500	Gate Valves - 3 in. through 48 inch for Water and Other Liquids
C502	Dry Barrel Fire Hydrants
C503	Wet Barrel Fire Hydrants
C504	Rubber-Seated Butterfly Valves
C506	Backflow Prevention Devices - Reduced Pressure Principle and Double Check Valve Types
C507	Ball Valves, Shaft - or Trunnion - Mounted - 6 in. through 48 inch - for Water Pressures up to 300 psi
C601	Disinfecting Water Mains
C602	Cement-Mortar Lining of Water Pipelines 4 in. (100 mm) and Larger in Place
C603	Installation of Asbestos - Cement Pipe

C900	Polyvinyl Chloride (PVC) Pressure Pipe, 4 inch through 12 inch, for Water
C950	Glass-Fiber-Reinforced Thermo- setting - Resin Pressure Pipe
M8	A Training Course in Water Distribution
M9	Installation of Concrete Pipe
M11	Steel Pipe - Design and Installation
National Fire Protection Association (NFPA), 470 Atlantic Avenue, Boston, Massachusetts 02210	
291	Marking of Hydrants
1963	Screw Threads and Gaskets for Fire Hose Connections



## APPENDIX B

### DISTRIBUTION SYSTEM HYDRAULIC ANALYSES

**B-1. General.** The sizing and location of mains, pump stations, and elevated storage facilities are dependent upon hydraulic analyses of the distribution system. The two major techniques used in analysis of distribution networks are reduction into equivalent pipes and Hard Cross Analysis. For all but the very smallest systems, these analyses are best performed on computers.

**B-2. Equivalent Pipes.** The equivalent pipe techniques is a means of reducing a complex pipe network into a simpler configuration. It involves the substitution of one pipe of specific diameter and variable length or specific length and variable diameter for a series of different size pipes or several parallel pipes, as long as there are no inputs or withdrawals of water between the end points of the system. Application of the equivalent pipe method is best demonstrated by example. Referring to figure B-1, assume that the pipe network shown is to be converted to an equivalent 8-inch pipe. The following procedure should be used.

*a. Series of different size pipes will be converted.* An example is ACD and ABD in part (A) of figure B-1 being converted to equivalent 8-inch pipes. A flow rate will be assumed through each branch, the resulting loss of head calculated through the branch, and the length of 8-inch pipe substituted, which will give the same total loss of head through each branch. For example, assume that 200 gpm flows through branch ACD and 400 gpm through ABD. Using tables or nomograms based on the Hazen-Williams formula, the loss of head through section AC is 1.51 feet per 1,000 feet of pipe length (assume  $C = 100$  for all pipes), so the total loss of head through pipe AC is  $(1.51/1000) \times 1000 = 1.51$  feet. Likewise, the loss of head through pipe CD at a flow of 200 gpm is 6.1 feet per 1,000 feet of pipe length, which gives a loss of head through CD of  $(6.1/1000) \times 800 = 4.9$  feet. Hence, the total loss of head through ACD is 6.4 feet. The length of 8-inch pipe which will have the same total loss of head at the same flow is  $6.4 \div (1.51/1000) = 4,240$  feet. The two pipes of branch ACD can be replaced by 4,240 feet of 8-inch pipe. The total loss of head through ABD at a flow of 400

$$\text{gpm is } (1.83 \times \frac{700}{1000}) + (0.75 \times \frac{2000}{1000}) = 2.78$$

feet. At the same flow of 400 gpm, an 8-inch pipe has a loss of head of 5.44 feet per 1,000 feet of length, so the length of 8-inch pipe equivalent to section ABD is  $(2.78/5.44) \times 1000 = 511$  feet. Part (B) of figure B-1 shows the configuration of the system after branches ACD and ABD have been converted to equivalent 8-inch pipes.

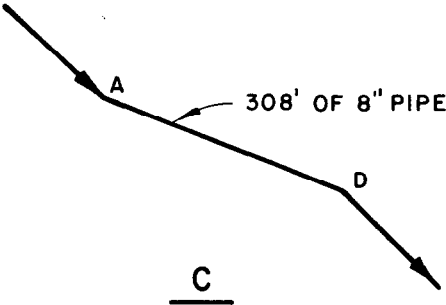
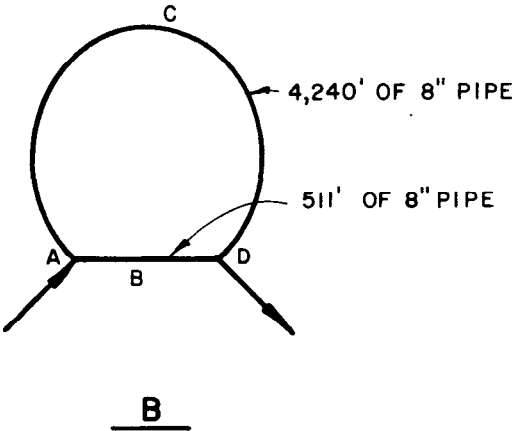
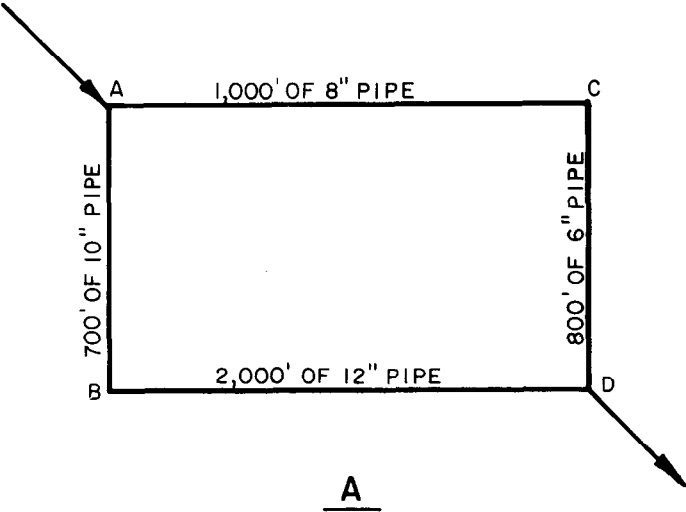
*b. The 8-inch equivalent pipes for ACD and ABD will be converted into a single equivalent 8-inch pipe.* Since it is known that water passing through ACD must have the same loss of head as water passing through ABD, a constant loss of head value can be assumed. For purposes of this example, a loss of head of 10 feet between A and D is arbitrarily chosen. At this total loss of head, the loss of head per 1,000 feet of length in ACD is 2.36 feet and in ABD is 19.6 feet. Referring again to nomographs or tables based on the Hazen-Williams equation, it can be determined that the flows producing these losses of head are 255 gpm in ACD and 800 gpm in ABD. Thus, the total flow from A to D with a loss of head of 10 feet is 1,055 gpm. At this total flow, the loss of head through a single 8-inch pipe is 32.5 feet per 1,000 feet of length. For a total loss of head of 10 feet from A to D at a total flow of 1,055 gpm, a single 8-inch pipe would be  $(10/32.5) \times 1000 = 308$  feet long. Part (C) of figure B-1 shows the single 8-inch pipe which is equivalent to section ABCD shown in part (A) of figure B-1.

#### B-3. Alternative equivalent pipe procedure.

Several variations of the equivalent pipe procedure are possible. The following is an alternative procedure for converting the pipe network of figure B-1 to a single equivalent 8-inch pipe, assuming that  $C = 100$  for all pipes.

*a. Arbitrarily select a rate of flow to be passed through both branches A, C, and D and branches A, B, and D. For this example, a flow of 0.5 mgd is used.*

*b. Calculate the losses of head through branches A, C, and D and branches A, B, and D.*



EQUIVALENT PIPE NETWORKS  
(NOT TO SCALE)

Figure B-1. Equivalent pipe networks.

Pipe	Diameter (inches)	Loss of Head Per 1,000 ft.	Length (feet)	Loss of Head (feet)
AC	8	4.18	1,000	4.18
CD	6	16.9	800	13.52
AB	10	1.41	700	0.987
BD	12	0.58	2,000	1.16

Loss of head through A, C, and D = 4.18 ft. + 13.52 = 17.70 ft.

Loss of head through A, B, and D = 0.987 ft. + 1.16 ft. = 2.147 ft.

c. Adjust the flow in branch ABD for the same loss of head as in branch ACD. This can be done with the following formula.

$$\frac{Q_2}{Q_1} = \left( \frac{HL_2}{HL_1} \right)^{0.54} \quad (\text{eq B-1})$$

where

$Q_1$  = initial flow in pipe

$Q_2$  = final flow in pipe

$HL_1$  = initial friction loss of head through the pipe

$HL_2$  = final friction loss of head through the pipe

Thus:

$$Q_2 = Q_1 \left( \frac{HL_2}{HL_1} \right)^{0.54} = 0.5 \left( \frac{17.7}{2.147} \right)^{0.54}$$

$$Q_2 = 1.56 \text{ mgd in ABD (loss of head} = 17.7 \text{ ft.)}$$

d. Find the total rate of flow through branches ABD and ACD with a loss of head of 17.7 feet in both branches. The total flow is equal to 1.56 mgd + 0.5 mgd = 2.06 mgd.

e. Determine the length of 8-inch pipe which will have a loss of head of 17.7 feet at a rate of flow of 2.06 mgd. At this rate of flow in an 8-inch pipe, the loss of head is 57.3 feet per thousand feet of pipe length. The total equivalent pipe length is:

Length of equivalent 8-inch pipe =

$$\frac{17.7}{57.3} \times 1,000 = 309 \text{ feet.}$$

**B-4. Hardy Cross analysis.** Equivalent pipe techniques can be used for finding flows or losses of head in simple systems, but more complex networks involving multiple withdrawal points and crossover pipes require different methods of solution. The Hardy Cross Analysis is one means of network analysis by which accurate determination of rates of

flow and losses of head through a system can be computed. It involves application of corrections to assumed values of flow or head until the system is in hydraulic balance. If flows are to be balanced, the correction factor to be applied to network flows is found by solving:

$$\Delta Q = - \frac{\sum H}{n \sum (H/Q)} \quad (\text{eq B-2})$$

where  $Q$  = flow in a particular pipe

$H$  = loss of head in that pipe

$n = 1.85$

In order to use the Hardy Cross Analysis, the following guidelines must be observed:

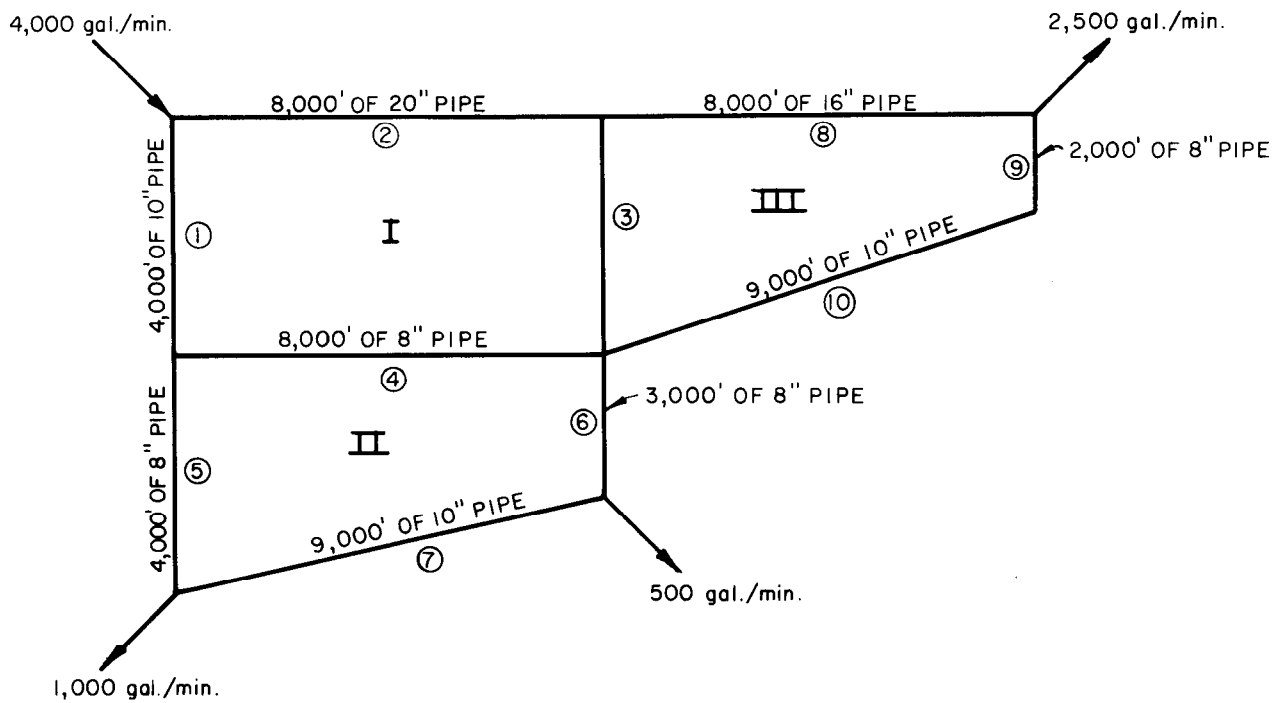
a. The configuration of the pipe network to be analyzed must be known or estimated. This includes pipe lengths, pipe diameters, and coefficients of roughness.

b. The locations and magnitudes of inflows and outflows to and from the system must be known or estimated.

c. Flows in either a clockwise or counterclockwise direction may be considered positive and those in the opposite direction will be negative. For example, if clockwise flows are assumed to be positive, counterclockwise flows will be negative. The same rule also applies for values of losses of head. Thus, the terms in the numerator of the above equation will always have the appropriate sign. The term in the denominator must always have a positive value because corresponding  $H$  and  $Q$  values have the same sign, therefore  $H/Q$  is always positive.

d. The sign of the calculated correction,  $\Delta Q$ , must be observed when modifying the flows in a pipe loop. Pipes appearing in more than one loop are subject to the combined corrections for the loops in which they appear. An example of the Hardy Cross Analysis is shown in figure B-2 and in table B-1. Figure B-2 gives the configuration of the pipe network and inflows and withdrawals from the network. The initial flow assumptions are shown in table B-2. All pipes are assumed to have a roughness coefficient of 100; final flow values are shown in table B-3.

**B-5. Other methods of hydraulic analysis.** Other hydraulic analysis techniques may be used if appropriate. Such techniques may include, but are not limited to, Newton Raphson network analysis and network simulation with analog computers.



EXAMPLE PIPE NETWORK FOR  
HARDY CROSS ANALYSIS  
(NOT TO SCALE)

Figure B-2. Example pipe network for Hardy Cross Analysis.



**Table B-1. Computations for Hardy Cross Analysis.****Trial 1**

Loop Number	Pipe Number	Pipe Diam. (in.)	Pipe Length (ft)	Initial Flow (gal/min)	Loss of Head (ft)	$\frac{H}{Q}$	$nS \frac{H}{Q}$	SH	DQ (gal/min)	Adjusted Flow (gal/min)
I	1	10	4,000	- 1,000	- 40.0	0.040			- 62	- 1,062
	2	20	8,000	+ 3,000	+ 21.0	0.007			- 62	+ 2,038
	3	8	4,000	+ 1,000	+ 118.4	0.118			- 388	+ 612
	4	8	8,000	- 500	- 65.6	0.131	0.548	33.8	+ 174	- 326
II	4	8	8,000	+ 500	+ 65.6	0.131			- 174	+ 326
	5	8	4,000	- 500	- 32.8	0.066			- 236	- 736
	6	8	3,000	+ 1,000	+ 88.8	0.089			- 236	+ 764
	7	10	9,000	+ 500	+ 25.0	0.050	0.622	146.6	- 236	+ 264
III	3	8	4,000	- 1,000	- 118.4	0.118			+ 388	- 612
	8	16	8,000	+ 2,000	+ 29.5	0.015			+ 326	+ 2,326
	9	8	2,000	- 500	- 16.4	0.033			+ 326	- 174
	10	10	9,000	- 500	- 25.0	0.050	0.400	- 130.3	+ 326	- 174

**Trial 2**

Loop Number	Pipe Number	Pipe Diam. (in.)	Pipe Length (ft)	Initial Flow (gal/min)	Loss of Head (ft)	$\frac{H}{Q}$	$nS \frac{H}{Q}$	SH	DQ (gal/min)	Adjusted Flow (gal/min)
I	1	10	4,000	- 1,062	- 44.8	0.042			+ 16	- 1,046
	2	20	8,000	+ 2,938	+ 20.3	0.007			+ 16	+ 2,954
	3	8	4,000	+ 612	+ 47.7	0.078			- 46	+ 566
	4	8	8,000	+ 326	- 29.8	0.091	0.403	- 6.6	+ 62	- 264
II	4	8	8,000	+ 326	+ 29.8	0.091			- 62	+ 264
	5	8	4,000	- 736	- 67.3	0.091			- 46	- 782
	6	8	3,000	+ 764	+ 54.0	0.071			- 46	+ 718
	7	10	9,000	+ 264	+ 7.7	0.029	0.522	+ 24.2	- 46	+ 218
III	3	8	4,000	- 612	- 47.7	0.078			+ 46	- 566
	8	16	8,000	+ 2,326	+ 38.8	0.017			+ 62	+ 2,388
	9	8	2,000	- 174	- 2.3	0.013			+ 62	- 112
	10	10	9,000	- 174	- 3.6	0.021	0.239	- 14.8	+ 62	- 112

Table B-1. Computations for Hardy Cross Analysis—Continued.

## Trial 3

Loop Number	Pipe Number	Pipe Diam. (in.)	Pipe Length (ft)	Initial Flow (gal/min)	Loss of Head (ft)	$\frac{H}{Q}$	$nS \frac{H}{Q}$	SH	DQ (gal/min)	Adjusted Flow (gal/min)
I	1	10	4,000	- 1,046	- 43.5	0.042			+ 5	- 1,041
	2	20	8,000	+ 2,954	+ 20.5	0.007			+ 5	+ 2,959
	3	8	4,000	+ 566	+ 41.1	0.073			- 8	+ 558
	4	8	8,000	- 264	- 20.1	0.076	0.366	- 2.0	+ 2	- 262
II	4	8	8,000	+ 264	+ 20.1	0.076			- 2	+ 262
	5	8	4,000	- 782	- 75.2	0.096			+ 3	- 779
	6	8	3,000	+ 718	+ 48.3	0.067			+ 3	+ 721
	7	10	9,000	+ 218	+ 5.4	0.025	0.488	- 1.4	+ 3	+ 221
III	3	8	4,000	- 566	- 41.1	0.073			+ 8	- 558
	8	16	8,000	+ 2,388	+ 40.9	0.017			+ 13	+ 2,401
	9	8	2,000	- 112	- 1.0	0.009			+ 13	- 99
	10	10	9,000	- 112	- 1.6	0.014	0.209	- 2.8	+ 13	- 99

## Trial 4

Loop Number	Pipe Number	Pipe Diam. (in.)	Pipe Length (ft)	Initial Flow (gal/min)	Loss of Head (ft)	$\frac{H}{Q}$	$nS \frac{H}{Q}$	SH	DQ (gal/min)	Adjusted Flow (gal/min)
I	1	10	4,000	- 1,041	- 43.2	0.041			+ 7	- 1,034
	2	20	8,000	+ 2,959	+ 20.5	0.007			+ 7	+ 2,966
	3	8	4,000	+ 558	+ 40.0	0.072			- 3	+ 561
	4	8	8,000	- 262	- 19.8	0.076	0.363	- 2.5	+ 5	- 257
II	4	8	8,000	+ 262	+ 19.8	0.076			- 5	+ 257
	5	8	4,000	- 779	- 74.7	0.096			+ 2	- 777
	6	8	3,000	+ 721	+ 48.6	0.067			+ 2	+ 723
	7	10	9,000	+ 221	+ 5.5	0.025	0.488	- 0.8	+ 2	+ 223
III	3	8	4,000	- 558	- 40.0	0.072			- 3	- 561
	8	16	8,000	+ 2,401	+ 41.3	0.017			+ 4	+ 2,405
	9	8	2,000	- 99	- 0.8	0.008			+ 4	- 95
	10	10	9,000	- 99	- 1.3	0.013	0.204	- 0.8	+ 4	- 95

**Table B-2. Initial flow assumptions**

Pipe number	Flow (gal/min)	Direction of flow
1	- 1,000	Counterclockwise
2	+ 3,000	Clockwise
3	+ 1,000	Clockwise (Loop 1)
4	- 500	Counterclockwise (Loop 1)
5	500	Counterclockwise
6	+ 1,000	Clockwise
7	+ 500	Clockwise
8	+ 2,000	Clockwise
9	- 500	Counterclockwise
10	- 500	Counterclockwise

**Table B-3. Final flow values**

Pipe number	Flow (gal/min)	Direction of flow
1	- 1,034	Counterclockwise
2	+ 2,966	Clockwise
3	+ 561	Clockwise (Loop 1)
4	- 257	Counterclockwise (Loop 1)
5	777	Counterclockwise
6	+ 723	Clockwise
7	+ 223	Clockwise
8	+ 2,405	Clockwise
9	- 95	Counterclockwise
10	- 95	Counterclockwise



## APPENDIX C

### THRUST RESTRAINT

**C-1. General requirements.** Thrust forces occur in watermains when the pipeline changes directions, stops, or changes size. On pipelines with unrestrained joints, as used in ductile iron pipe installations, thrust blocks or restrained joints are required. For welded steel pipelines, flanged joints and lugged joints in concrete and ductile iron pipelines, other forms of anchorage are not usually required. All thrust anchorages shall be designed for a safety factor of not less than 1.5 under maximum pressure loading. The magnitude of hydrostatic thrust may be determined from the following formula:

$$\text{At bends: } T = 2 \pi r^2 p \sin \frac{\Delta}{2} \quad (\text{eq C-1})$$

$$\text{At dead end or branch: } T = \pi r^2 p$$

where:

$T$  = thrust in pounds

$r$  = radius of pipe joints in inches

$p$  = water pressure in psi

$\Delta$  = bend deflection angle

**C-2. Thrust blocks.** Thrust block size is calculated based on the bearing capacity of the soil:

$$\text{Area of block} = L \times D = \frac{T}{a} (\text{F.S.}) \quad (\text{eq C-2})$$

where:

$L$  = length of block in feet

$D$  = depth of block in feet

$T$  = thrust in pounds

$a$  = safe bearing value for soil in psf

F.S. = factor of safety

**EXAMPLE C-1.** Calculate the thrust block bearing area required for a 12-inch pipe of a 90-degree bend, internal pressure of 120 psi including surge, allowable soil bearing pressure of 3000 psf.

Solution:

$$T = 2 (3.14) (36) (120) (\sin 45) \quad (\text{eq C-1})$$

$$T = 19,180 \text{ lbs}$$

$$L \times D = \frac{19180}{3000} \quad (1.5) \quad (\text{eq C-2})$$

$$L \times D = 9.59 \text{ s.f.}$$

$$\text{for } D = 2.5 \text{ ft., } L = 3.8 \text{ ft.}$$

Design of thrust blocks for vertical bends is the same as for horizontal bend. For top bends, the block must be sized to adequately resist the vertical component of thrust with the dead weight of the block, bend, water in the bend and overburden. Steel straps are used to tie the pipe to the thrust block when the block is placed below the pipe, and reinforcing steel may be necessary to resist tensile forces within the block.

**C-3. Restrained joints.** Restrained joints are used as an alternate to thrust blocks to avoid uncertainties such as excavation behind a block. For ductile iron pipe, the length to be restrained is calculated as follows:

$$L = \frac{4 \pi r^2 p (\text{F.S.}) \tan \frac{\Delta}{2}}{(2f \tan \frac{\Delta}{2}) + (D s)} \quad (\text{eq C-3})$$

where:

$L$  = length to be restrained on each side of bend in feet

$r$  = radius of pipe in inches

$p$  = water pressure in psi

F.S. = factor of safety

$\Delta$  = bend deflection angle

$D$  = pipe outside diameter in feet

$f$  = conduit frictional resistance in plf

$s$  = passive soil pressure in psf

Conduit frictional resistance is calculated from the following equation, using values in Table C-1 and table C-2.

$$f = A f_c C_s + \pi w \text{ RHD} \tan (f_p \Delta) \quad (\text{eq C-4})$$

where:

$A$  = conduit surface area in sq. ft. per lin. ft.

$f_c$  = ratio of pipe cohesion to soil cohesion

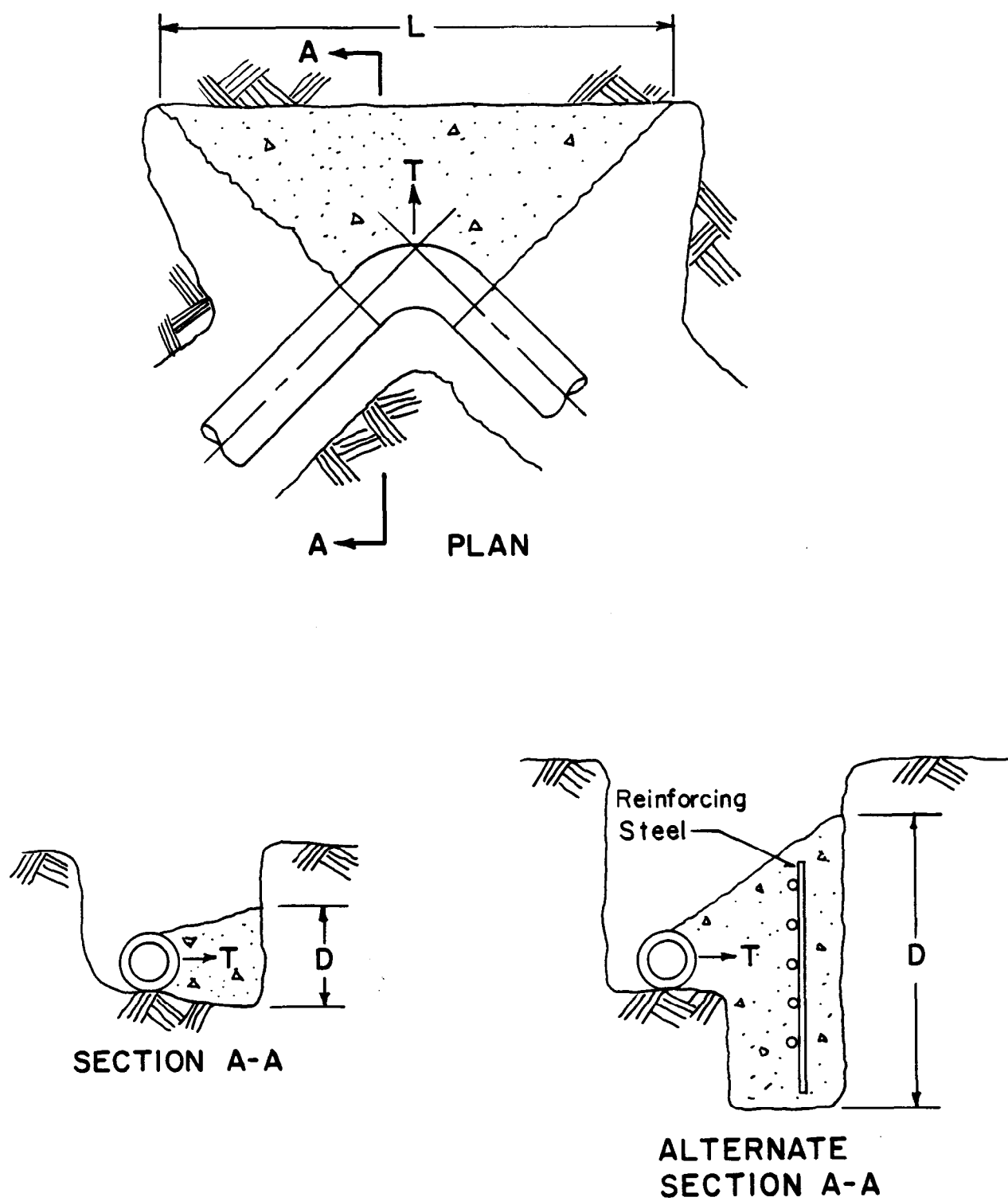


Figure C-1. Typical thrust blocking.

$C_s$  = soil cohesion in psf  
 $w$  = soil unit weight in pcf  
 $R$  = reduction factor  
 $H$  = cover above conduit in feet  
 $f_p$  = ratio of pipe friction angle to soil friction angle

Passive soil pressure is calculated according to the Rankine Theory by the following equations:

$$s = w \left( H + \frac{D}{2} \right) N + 2 C_s \sqrt{N} \quad (\text{eq C-5})$$

$$N = \tan^2 \left( 45^\circ + \frac{\Delta}{2} \right) \quad (\text{eq C-6})$$

**EXAMPLE C-2.** Calculate the restrained length for a 24-inch ductile iron pipe.

Given:  $D$  = 2.15 feet  
 $r$  = 12.52 inch  
 $p$  = 250 psi  
 F.S. = 1.5  
 $\Delta$  = 45 degrees  
 $w$  = 110 pcf

$H$  = 3.5 feet

$C_s$  = 500 psf

Cohesion moist granular soil, friction angle of 17 degrees, from Table C-1:

$f_p$  = 0.65

$f_c$  = 0.35

Solution:

$$N = \tan^2 (45 + 22.5) = 5.83 \quad (\text{eq C-6})$$

$$s = (110)(3.5 + 1.1)(5.83) + (2)(500)(2.41) = 2950 + 2410 = 5360 \text{ psf} \quad (\text{eq C-5})$$

$$A = \pi D = (3.14)(2.15) = 6.75 \text{ sq. ft./lin. ft.}$$

$$f = \frac{(6.75)(0.35)(500) + (3.14)(110)(.75)(3.5)(2.15)}{\tan (0.65)(45)} = 1181 + 2273 \text{ plf}$$

$$L = \frac{(4)(3.14)(157)(250)(1.5)(.414)}{(4)(2273)(.414) + (2.15)(5360)} = 20.0 \text{ feet} \quad (\text{eq C-3})$$

For normal pipe lengths of 18 feet, restrain two joints on each side of bend.

**Table C-1. Soil friction and cohesion factors**

Soil Description	Friction Angles (degrees)	$C_s$	$f_p$	$f_c$
Sand, dry well graded	44.5	0	0.76	0
Sand, saturated, well graded	39	0	0.80	0
Silt, dry, passing 200 sieve	40	0	0.95	0
Silt, saturated, passing 200 sieve	32	0	0.75	0
Cohesive granular soil wet to moist	13 - 22	385-920	0.65	0.35
Clay, wet to moist at maximum compaction	11.5 - 16.5	460-1,175	0.50	0.50
			0.50	0.80

**Table C-2. Reduction factors**

Existing Conditions	Reduction Factor
General Construction - backfill soils compacted to critical void ratio	2/3
Well compacted backfill and selected backfill	3/4
Shallow cover - H less than one half of D	1/2





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